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General Report for Theme One - Foundations for Structures and Failure Records

Robert C. Kirby
Woodward-Clyde Consultants, Wayne, New Jersey

Guilaine Roussel
Woodward-Clyde Consultants, Wayne, New Jersey

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INTRODUCTION

Fifty-two papers and two special lectures are included within Theme One: Foundations for Structures and Failure Records. These case histories represent a wealth of experience and the Reporters decided to discuss the papers in the context of questions asked by practicing foundation engineers on virtually every project. These questions and the general issues that are related to them are as follows:

Question	Issue
• Has the program of field investigations and laboratory testing identified all the important site characteristics?	• Unexpected Subsurface Conditions
• Are soil conditions unusual or is experience with such soil conditions very limited?	• Special Soils
• Should methods be considered for improvement of the engineering properties of the soils?	• Soil Improvement
• How sensitive is the structure to foundation movements and how confident are we that the actual performance of the foundation will meet performance requirements?	• Performance of Foundations: - predicted - measured - allowable
• What are the effects on adjacent structures?	• Adjacent Structures
• Is there a need to consider innovative foundation systems to improve performance and/or reduce costs?	• Innovative Foundation Systems

Figure 1 indicates how the papers and special lectures included within Theme One are related to these general issues. This General Report discusses those aspects of the papers and special lectures that are related to each of the issues identified above. Papers that cover several issues are discussed more than once.

UNEXPECTED SUBSURFACE CONDITIONS

Serious problems with foundations can result from subsurface conditions that are not discovered during the site investigation and/or are not considered during foundation design. These unexpected subsurface conditions can be attributed to various causes: (a) the field investigation was simply inadequate, (b) important conditions affecting the project existed outside the depth and the area normally considered during a field exploration, (c) minor geologic details within the depth or area of exploration were overlooked, or (d) special problems developed because of soil conditions that were known to exist but not known to cause problems. As indicated in Figure 1, fourteen papers submitted under Theme One provide relevant examples of projects where problems developed because of unexpected subsurface conditions.

Chen and Heller present a case history where an inadequate initial field investigation led to excessive settlement and cracking of a service water pumphouse and intake structure for a nuclear power plant. The pumphouse and intake structure were constructed on compacted fill overlying residual soils. The compressibility of the residual soils was investigated at a location more than 1000 ft away from the final location of the structure. This approach did not provide reliable information because of variability of the residual soils. The Authors report that the problem was compounded by the placement and compaction of fill (consisting of excavated residual soils) at a water content that was too wet of the optimum water content. Remedial measures included preloading of the subsurface materials to accelerate settlements and grouting of the structural cracks after removal of the preload.

Engeling, Hayden and Hawkins report that inadequate sampling of subsurface material during an initial exploration program misled the designers in their selection of pile driving criteria. The project consisted of driving concrete cylinder piles for the support of a pipeline trestle in the Arabian Gulf. The use of driven samplers during the initial subsurface investigation led to imprecise characterization of the strength of lightly-cemented carbonate materials that, in turn, resulted in unexpected pile behavior during driving. Extensive investigation was undertaken during pile driving using cored samples of the materials for accurate determination of soil and rock characteristics. Continuous engineering supervision during construction, modification of installation procedures as better subsurface information and performance data became available, and an extensive pile load testing program led to the successful installation of 1500 piles.

Another case where insufficient subsurface exploration led to problems is presented by Peaker. The initial investigation for a fourteen-story building did not identify the presence of a layer of soft highly-compressible clay in the foundation

Paper or Lecture Number and Authors	GENERAL ISSUES Unexpected Subsurface Conditions	Special Soils	Soil Improvement	Performance of Foundations	Adjacent Structures	Innovative Foundation Systems	Other Issues
Special Lecture: T. Iwasaki				•			•
Special Lecture: S. Hansbo				•		•	
101 N. Kumapley and S. Ramachandra	•	•					
104 Y. Barton, R. Parry, and W. Liam Finn				•			
105 K. Peaker	•		•				
108 S. Bandyopadhyay and R. Reuss	•	•					
110 R. Bhandari, M. Soneja, and D. Sharma				•			
111 A. Stipho		•					
114 M. Luong	•				•		
115 Y. Xu, Y. Liu, Y. Shi, and S. Xin				•			
116 G. Ranjan, S. Prakash, S. Saran, and B. Singh							•
117 E. Winter and P. Chung				•			
119 S. Saye				•			
121 D. Lane				•			
122 B. McClelland and E. Ulrich	•		•				
123 L Ozaydin and S. Inan				•			
126 A. Butcher and A. Marsland		•		•			
127 G. Blight		•		•		•	
129 C. Mastrantuono, A. Tomiolo, and E. Arcangeli	•						
130 D. LaGatta and T. Keller				•			
131 E. DeBeer, M. Wallays, and E. Goelen				•			
133 D. Bhargava, D. Nath, S. Kapoor, and S. Singh	•						•
136 I. Khan		•					
139 J. Dugan and D. Freed					•		
140 B. Tan	•						
141 F. Leon			•				
142 Y. Zhang				•			
143 S. Gazioglu and J. Withiam	•			•			
146 L. Wilson, M. Stomer, and P. Girault		•	•			•	
147 T. Kaderabeck, D. Barreiro, and M. Call				•			
148 C. Clayton, J. Milititsky, and L. Carvalho				•			
149 S. Abo-El Magd, H. Hosny, and M. Mashhour				•			
151 Y. Wang and J. Yuan						•	
153 C. Sheng							•
154 R. Wei				•			
155 M. El-Sohby and O. Mazen					•		
157 R. Olson, N. Dennis, and D. Winter				•			
158 A. Lutenegger, B. Remmes, and L. Handfelt		•		•			
161 S. Handa						•	
162 A. Ciancia and H. Horn	•				•		
163 K. Chung and L. Cundy				•			
164 F. Newman and A. DiGioia						•	
165 G. Bauer				•		•	
166 G. Felio and G. Bauer				•		•	
168 H. Taylor and A. Joseph	•						•
171 P. Engeling, R. Hayden, and R. Hawkins	•			•			
174 R. Fallgren and C. McClure	•	•					
176 J. Chen and L. Heller	•						
179 S. Ahmed			•	•			
180 M. Jamiolkowski and R. Lancellotta							•
181 A. Arcones and A. Soriano			•	•			
182 J. Briaud, A. Pacal, and A. Shively				•			
184 S. Lee and S. Sithichaikasem				•			
185 A. Chummar		•			•		

FIGURE 1 - RELATIONSHIP BETWEEN THEME ONE PAPERS AND SOME IMPORTANT ISSUES FOR FOUNDATION ENGINEERS

materials because of insufficient sampling. The existence of this layer, combined with a change in the design of the foundation system from piles to footings on improved ground, led to very large settlements. Legal disputes following the identification of the settlement problem resulted in the demolition of the building.

Kumapley and Ramachandra report severe distress in a four-story reinforced concrete building supported on strip footings because of differential heave of the foundation material. The building was founded on fresh and decomposed clay shale but a channel deposit of permeable gravel in an expansive clay matrix was present beneath a portion of the building. Both the decomposed clay shale and the channel deposit are reported to exhibit expansive characteristics. The subsurface investigation performed after development of the problem suggested that the differential heave of the foundation occurred as a result of saturation of the channel deposit because of poorly maintained surface drainage facilities. The Authors postulate that it is likely that no subsurface exploration was performed prior to construction of the building and that the existence of the channel deposit was unknown. Successful remedial measures are reported to have consisted of replacement and underpinning of certain footings and improvement of surface drainage.

McClelland and Ulrich describe differential settlement and tilt that occurred during the construction of an eighteen-story building in Florida founded on a pile-supported mat. The initial subsurface exploration extended to a depth of 100 ft and encountered hydraulic sand fill overlying peat and a limerock formation with sand layers. Further investigation after the development of the settlement revealed the existence of a 25-ft zone of very loose calcareous sand below a depth of 125 ft, that was interpreted as a partially filled cavity in the limerock. The Authors report successful stabilization of the building by extensive grouting of the limerock. This paper provides a good example of a deep subsurface problem that was undetected because it was beyond the depth of an exploration program that would normally be considered adequate for the type of building described. Another example of such a problem is described by Taylor and Joseph for a powerhouse located at the bottom of a mountain slope. The powerhouse was founded on dense glacial till overlying clayey silt that, in turn, is underlain by silt, sand, and gravel. The initial subsurface investigation extended to a depth of 80 ft to 100 ft, not deep enough to detect the artisian pressure in the silt, sand, and gravel aquifer. Extensive ground cracking and movement, as well as cracking of the concrete penstock tunnels of the pumphouse, occurred during the first spring following completion of construction. Detailed investigation is reported to have shown that these movements were directly related to high artesian pressure developing under the relatively impervious clayey silt layer. Remedial measures included construction of a stabilizing fill and installation of a relief well system to control the artesian pressure.

An example of unexpected subsurface conditions outside the project site that had detrimental effects is provided by Luong who describes the severe damage caused to a twenty-year-old five-story building supported on spread footings on soils containing gypsum. The Author shows that pumping for dewatering of a nearby sewer excavation led to groundwater movement in the building subsoils and to dissolution of the gypsum, resulting in significant differential settlement and cracking of the building. Movement and cracking are reported to have stopped when dewatering was stopped.

Fallgren and McClure present the case of a nuclear power plant where a minor geologic characteristic of the foundation stratum was overlooked, leading to unexpected behavior of the foundation during construction. The power plant is founded on clayey limestone and calcarous claystone overlain by gravel and silt with veins of gypsum and anhydrite. During excavation for the power plant, significant heave of the foundation was experienced. The foundation heave was larger than expected from normal elastic rebound and took place over a long period of time. Further investigation and laboratory testing of the rock revealed that a small amount of smectite (3 percent) - a clay mineral susceptible to expansive characteristics - was disseminated through the rock mass. This mineral is reported by the Authors to be the cause of the significant heave observed. Remedial measures were aimed at controlling the amount of water that could penetrate the rock and included surface drainage and sealing, installation of a deep drainage system, and construction of a compacted silt blanket and slurry wall. Tan presents a number of case histories in Malaysia where geologic characteristics of foundation soils and rocks were overlooked and led to construction problems, delays, and significant cost increases. The various examples provided emphasize the need for careful consideration of geology to provide insight into the possible behavior of foundation materials.

Several papers are included in Theme One that deal with special problems that developed because of unexpected events or special conditions that were not fully understood during the design phase. Bandyopadhyay and Reuss present two cases where abrupt changes in soil moisture had significant effect on building foundations. The first case involves a two-story building and a parking lot on stiff highly plastic expansive clays. Significant cracking and floor heave occurred in the building when a nearby water line broke and the clay became saturated. The Authors also report long-term deformations and distress of the pavement of the parking lot over many years around planted areas. The Authors attribute this phenomenon to the shrinkage of the clay because of withdrawal of moisture by vegetation.

The second case considered by Bandyopadhyay and Reuss considers settlement of a basement floor and a sidewalk adjacent to a multi-story building after the rupture of a water line under the building. The Authors conclude that water from the broken line washed out sand, gravel, and fines from under the basement slab and from the perimeter drain located under the sidewalk. Voids appeared under the basement wall, as well as under the sidewalk resulting in settlements. The Authors indicate that relatively simple, straightforward remedial measures for the two cases were developed, but no information regarding the implementation or performance of these measures is provided.

Bhargava, Nath, Kapoor and Singh report a case where a powerhouse and power channel of a hydroelectric power plant were founded on clay shale interspersed with pervious bands of coarse sand, gravel, and thin seams of plastic clay. These clay seams daylighted on the slopes of the power channel and the powerhouse excavations and caused a number of major slides during the rainy seasons. Stabilizing measures included flattening of the slopes, addition of filters and rock toes, and relief wells. The Authors conclude that regular monitoring of the structure should be carried out because the clay seams remain a potential problem for the long-term performance of the power plant.

Gazioglu and Withiam present the case of a 300-ft-diameter floating roof tank constructed on a thick deposit of compressible recent alluvial materials overlying stiff silty clay and dense fine sand. The potential for large settlements of the tank led to the controlled water loading of the tank prior to its placement into service. During this preliminary loading, the tank experienced differential settlements. The Authors report that further subsurface investigation led to the conclusion that the movements of the tank had probably been caused by the existence of a thicker layer of normally consolidated clay and silt under a portion of the tank. This thicker layer of compressible material is believed to correspond to an old filled meander of the Mississippi River. Proposed remedial measures included leveling of the tank by mudjacking and preloading. Mastrantuono, Tomiolo, and Arcangeli describe differential settlements experienced by a six-story building constructed on 120 ft of soft organic clay. In this case, remedial measures involved differential loading of the foundation and the use of sand drains in an attempt to reduce differential movements.

Finally, the last paper that falls into this category of unexpected subsurface conditions shows that, although unexpected subsurface conditions will always be a problem to the foundation engineer, careful planning and monitoring of construction could significantly reduce the risk of detrimental effect of these conditions. Ciancia and Horn describe the case of a large excavation in rock in a crowded city environment. The approach selected on this project included an extensive field exploration program to obtain both geologic and geotechnical data, a conservative design of a temporary support system, and the installation of an extensive instrumentation system. The instrumentation was monitored throughout excavation and construction and was aimed at detecting movement at an early stage so that remedial measures could be implemented before any detrimental effect was experienced by the nearby structures. This approach led to a successful project under difficult conditions.

SPECIAL SOILS

The foundation engineer is sometimes faced with special soils within the foundation bearing strata that can be the source of serious problems if not properly treated. These special soils exhibit unusual behavior that needs to be considered when choosing a foundation system or that requires special precautions. Some of these soils are specific to certain regions of the world and not always well understood; others are widely spread throughout the world, but sometimes overlooked when designing the foundation system.

As shown in Figure 1, ten papers dealing with special soils and rocks are included under Theme One. Five of these papers are concerned with expansive clays. These clays, if not properly treated, can lead to serious distress in buildings by exerting large swelling pressure when saturated and by generating significant foundation heave. Kumapley and Ramachandra describe the serious distress caused to a four-story reinforced concrete framed building by differential foundation heave due to saturation of underlying expansive clay exposed to water from a faulty surface drainage system. Bandyopadhyay and Reuss discuss damage to a two-story building because of saturation and swelling of expansive clay upon rupture of a water line. They also describe pavement distress upon shrinkage of expansive clay because of moisture withdrawal by vegetation. Fallgren and McClure report unusual heave of a large excavation for a nuclear power plant because of the presence of a small amount of smectite - an expansive clay mineral - dispersed through the underlying clayey limestone and calcarous claystone. Chummar reports settlements up to 20 cm and subsequent cracks experienced by a three-story residential building constructed on 4.5 m of expansive sandy clay. The Author concludes that the settlement was caused by shrinkage of the clay when the water table moved from 3 m above to 3 m below the bottom of the clay layer during a drought period. Recommended remedial measures included lime treatment of the clay to ıcrease its shrinkage limit and structural reinforcement of e building. No performance record of these remedial asures is provided.

Blight presents a case where the existence of swelling soil was identified during the subsurface investigation and th design of a power station foundation was tailored to addres the potential problems caused by these soils. The site o the power station is underlain by horizontally bedde sedimentary rocks and is crossed by an old buried rive channel. The near-surface materials are residual soils ove a portion of the site and alluvium over the area of the ol river channel. The residual soils are primarily stiff to ver stiff fissured clayey silts and the alluvium is very variabl with layers and lenses of clean sands and clayey sands. Th groundwater table in the residual soils is shown to vary fror 11 m to 20 m below grade. Groundwater in the area of th old river channel is reported to drop by up to 20 m and t be below the bottom of the alluvium. The Author report that the soils located above the water table are desiccate and he expects that moisture will be returned to these soil upon clearing of the vegetation and that this will result i significant heave. The Author identifies this expected heav as a potential problem for piles to be installed at the sit because they will be subjected to uplift forces. He describe how the design of the power station foundation wa approached in an attempt to alleviate the expected problem associated with soil heave. The expected amount and rat of heave was estimated based on laboratory determine parameters and on assumed recharge of the aquifer. Th expected uplift forces exerted on the piles were evaluate using the estimated amount and rate of heave, as well a large-scale field plug pulling tests. These tests were als used to evaluate the effectiveness of vermiculite filled sleeve in reducing uplift forces on piles. Based on these predictior and on the result of the tests, a vermiculite sleeve pile wa designed to reduce uplift forces and is reported to have bee widely used at the site. Ducts and cooling towers have bee supported on piles and voids have been provided under th pile caps to accommodate the expected soil heave. No dat are yet available to verify the performance of this innovativ foundation system.

Butcher and Marsland present a case of a bridge abutmer constructed on fractured chalk. The Authors point out tha although the thick chalk deposits of Southeast England ar very uniform in composition, the chalk can behave quit differently from one site to another because of loca phenomena, such as previous loading, erosion, and weathering In the case history reported, the broken chalk was found t have a compressibility five to twenty times larger than th chalk of the same deposit with a similar visual classificatior but located about 100 km away. The Authors attribute thi difference in characteristics to the variation in the degre to which the fractures in the rock mass are partially oper They caution against predictions of behavior based on observe behavior of similar grading chalk at other sites. In the cas history reported, the compressibility of the chalk wa measured in-situ, in large-scale plate loading tests and th results of these tests were used to design the bridge abutmer foundations. An extensive instrumentation program wa implemented to verify the satisfactory performance of th structure.

Wilson, Stomer and Girault discuss the unusual case of hospital complex on a composite foundation of basaltic lav and coarse sand fill with lava fragments. The Authors repor that loose coarse sand with lava fragments up to 7.5 m i thickness overlies basaltic lava in places. The lava was foun to be 12 m to 14 m thick and to overlie hard Tertiary cla and silt. The situation is complicated by the fact that th lava, because of its origin, contains a number of cavities an ash-filled inclusions that are prone to collapse under loac The Authors report that a foundation system consisting c spread footings supported on both lava and densified coars sand was designed and constructed for the complex.

Stipho provides an overview of the special subsurface conditions and foundation problems in the desert regions of the Middle East. The major features that are likely to affect foundation engineering in the area include cavities in carbonate rocks, weakly-cemented sands and gravels that have a potential for collapse when exposed to moisture, expansive soils, and salt bearing soils that are very corrosive. Dunes of loose sand that are moved by the wind are also a serious problem for maintenance of highways and railroads. The Author points out the general instability of the soil profile in these desert regions because of the severity of the climate and the intensity of the weathering process. He emphasizes the need for site-specific investigations and for flexibility in design and construction to handle unexpected local conditions that are the result of the extreme variability of the subsurface conditions in the area.

Khan discusses construction on Sabkha soils in Libya. These soils result from artificial filling of lagoons over long periods of time and are characterized by high salt and chemical concentrations that result from seasonal moisture variations. The Author reports seven years of settlement measurements for a building founded on a soft deposit of Sabkha soils. These measurements suggest that differential settlements were large and led to significant tilt of the building. Unfortunately, no data on settlement immediately after construction are available and this makes it difficult to develop a conclusive explanation for the cause of the settlements. The Author closes the paper with some general recommendations for construction on Sabkha soils based on the experience of local contractors.

The last paper that falls into the category of special soils deals with unsaturated loess. Lutenegger, Remmes, and Handfelt discuss the potential for unsaturated loess to collapse upon wetting leading to unacceptable settlements, and as a result, forcing the foundation engineer to use deep foundations to bypass the problem layer. However, the Authors suggest that when the conditions are such that the potential for wetting of the loess does not exist, serious consideration should be given to more economical shallow foundations. They discuss the use of various techniques to predict the settlements of the unsaturated loess under load, using the case of a large standpipe supported on a mat foundation to evaluate the accuracy of the various prediction methods.

SOIL IMPROVEMENT

Improvement of in-situ foundation soils is an approach that is receiving increasing attention from the foundation engineer because it can result in signficant savings in overall cost for foundations. This approach often allows the use of shallow foundations at sites that would otherwise have required deep foundations.

Figure 1 identifies six papers presented under Theme One that deal with various soil improvement methods. Leon provides an extensive description of the Dynamic Precompression Treatment of soils, also well known as Dynamic Compaction, that consists of the repeated lifting and free dropping onto the ground surface of a relatively heavy weight from a great height. The Author proposes a number of parameters that can be used, based on his experience, for the design of a treatment program, as well as a simple method for evaluation of the performance of the treatment. He also suggests guidelines for the control of vibrations generated by the Dynamic Precompression Treatment that are of special importance for neighborhood buildings. The Author describes an unusual in-situ testing technique to evaluate soil compressibility at various depths before and after treatment. This testing technique consists of using a 2.84-inch-diameter plate lowered into a lined borehole to perform miniature plate load tests at various depths. The Author reports extensive experience with this testing technique and suggests that the results obtained from these tests provide a better evaluation of the effectiveness of the treatment than other commonly used techniques.

Leon discusses the case of an eleven-story tower supported by footings on a deep deposit of loose to medium dense fine silica sand intermixed with shelly calcareous sand that was densified by Dynamic Precompression. Three similar eleven-story towers had been successfully constructed on spread footings on the same subsurface material. However, during construction of the fourth tower, the developer decided to investigate the possibility of increasing the height to fifteen stories without modification to the foundation system. The Author shows how the parameters he proposes to use for the design and evaluation of Dynamic Precompression Treatment could be used for this specific case and how the performance records of the other three towers were used to predict settlement, as well as determine the allowable soil bearing pressure for the new tower. The results of this evaluation led to the conclusion that the tower could be constructed to fifteen stories without modification to the foundation system. Leon reports that the tower was successfully completed and that measured settlements were approximately 80 percent of the predicted values.

Wilson, Stomer and Girault describe another case of successful use of dynamic compaction for the construction of a hospital complex. The high rise portions of a hospital complex were supported on footings carried down to a layer of basaltic lava, while the low rise portions of the complex were supported on a layer of loose coarse sand up to 7.5 m in thickness that overlaid the lava. The sand was successfully densified by dynamic compaction to attain an allowable bearing pressure of 30 tons per square meter. The effectiveness of the dynamic compaction treatment was evaluated by plate load tests. The complex was successfully constructed. The maximum recorded settlement of any foundation member was 20 mm, which is slightly higher than that predicted using the plate load test data.

Arcones and Soriano describe the use of vibro-compaction to improve soil density to a depth of about 12 m at a power plant constructed on a deep deposit of loose to medium dense sand over 50 m in thickness. Major problems associated with the subsurface conditions were settlement of shallow foundations and liquefaction potential under ground accelerations on the order of 0.10 g to 0.15 g. The Authors report that the sand was successfully densified using the vibrocompaction method that consisted of densifying the soil by displacing columns of sand with a combination of vibration and water jet and replacing the displaced material by gravel, thereby creating a gravel column. A total of 660,000 m^3 of sand were treated to a depth of about 12 m using this method. The relative density of the sand is reported to have been improved by more than twenty percent and the risk of liquefaction to have been removed. The soil improvement allowed the use of spread footings for light structures and small precast floating piles for heavier structures. The Authors draw some conclusions of general interest related to the spacing and distribution of treatment points and to the consumption of filling materials, and also identify the fact that the treatment method tends to accentuate the heterogeneity of the soils treated.

Peaker discusses a case history where vibrocompaction did not lead to a successful improvement of the soil. The original foundation design of a fourteen-story building to be supported on piles or caissons was modified to a system of spread footings on sand, improved by vibrocompaction. The original subsurface investigation revealed that the site was underlain by 4 m to 6 m of compact to dense silty sand over bedrock. It is reported that the effectiveness of the vibrocompaction treatment was verified at the beginning of the project by a soil consultant and the spacing of the stone columns was adjusted. After completion of the soil treatment, construction of the building proceeded. The Author reports that settlement started virtually as construction began and had reached values in excess of 400 mm on one side of the building four years after construction. Additional subsurface investigation revealed the existence of a soft silty clay layer up to 2.8 m in thickness, within the sand. The Author states that no adequate construction records were kept during the implementation of the vibrocompaction treatment and implies that no engineering inspection of field operations was provided. After pointing out that the observed settlements were in excess of what could be anticipated if the subsurface material had not been treated, Peaker postulates that the vibrocompaction treatment disturbed the silty clay layer that later became overstressed. It it unfortunate that the soft clay layer which was overlooked during the site investigation was not identified during the implementation of the vibrocompaction treatment. The Reporters feel that field inspection of the operations by a qualified geotechnical engineer would have probably allowed early detection of the unexpected subsurface conditions and may have alleviated the problems experienced by the building.

One paper was included in Theme One that deals with the improvement of cohesive materials. Ahmed discusses the construction of a tank farm. The subsurface conditions consist of approximately 7 ft of sand fill, overlying 9 ft of firm clay and 11 ft of soft organic clay. The deeper materials are soft to firm clay and overconsolidated deposits of sand and clay. The tanks apply a total contact pressure that is twenty-five percent larger than the allowable bearing pressure as revealed by the subsurface investigation. The Author reports that a program of soil improvement was undertaken to accelerate consolidation of the clay and increase its strength. The program included installation of Alidrains (wick drains) in a peripheral band under the edge of the tanks, preloading of the tank sites, construction of the tank on 5 ft of areal fill, placement of counterbalancing berms around the periphery of the tanks, and stage loading of the tanks with water. Pore pressures, settlements and lateral displacements of the soil were monitored, as well as response of the tanks. The Author reports that the soil improvement program allowed successful construction of the tanks without the high cost of pile foundations.

Another type of soil improvement is described by McClelland and Ulrich as a remedial measure for deep seated foundation settlements. An eighteen-story reinforced concrete building on a pile-supported mat foundation suffered large settlement and tilt because of the existence of a loosely filled cavity in the limestone rock underlying the site. This cavity was located at a depth of 125 ft, at least 60 ft below the lowest level of pile tips. The Authors describe an extensive program that was undertaken to grout the loose material within the cavity. Cement grout was injected from the bottom up through gun-perforated grout pipes. The Authors report that the grouting initially accelerated the building settlements, but finally stabilized them to an acceptable relatively uniform rate of 0.0003 ft per day. Eleven years of settlement monitoring revealed that the settlement rate continued to decrease, the total settlement at the end of the monitoring period was 4.1 inches to 8.3 inches. McClelland and Ulrich note that the building has been successfully in use for nineteen years and has withstood hurricanes.

PERFORMANCE OF FOUNDATIONS: PREDICTED, MEASURED, AND ALLOWABLE

The foundation engineer's basic assignment is to develop economical foundation system that will meet the performan requirements of the structure. In most cases, the performance requirements are simply that the movements the structure should not interfere with its intended use r cause architectural or structural damage. The foundati engineer then translates this general requirements ir allowable total and differential displacements of t foundation, predicts the displacement of the foundati system, and modifies the foundation design until t movements are less than allowable. It is also necessary f him to check that the factor of safety against very lar movements is adequate.

Many of the case histories included in Theme One provi measured foundation movements, compare measured predicted performance, and/or discuss the consequence to t structure of the measured movements. Figure 1 identif the papers that are related to the performance of foundatio The report that follows groups the relevant papers into thr categories: footings and rafts, deep foundations, and tar and silos. The latter category is singled out because t allowable movements are generally large. The special lectu by Iwasaki considers the performance of bridges duri earthquakes in Japan. This paper will be considered first a special case.

Performance of Bridges During Earthquakes

Major earthquakes in Japan have destroyed 29 bridges a damaged over 3000 bridges during the past 60 years. Iwasak special lecture describes the behavior of bridges in Jap during eight major earthquakes since 1923. The spec lecture describes the ground motion for each of the eig earthquakes, presents an overview of the general dama caused by the earthquake, and then considers in some det the performance of one or two bridges during each of t eight earthquakes. Iwasaki concludes that seismic dama to bridge structures are generally caused by the lack resistance at bearing points, substructures, and surround soils. As a result of the weakness of these portio substructures have moved excessively and superstructu have experienced large movements, or fallen down. Iwas notes that correct assessment of the magnitude of the des seismic force is most important. In addition, it is importa to give special attention to topographical and geologi characteristics of the bridge sites, to evaluate geotechni issues such as liquefaction, to develop design details prevent bridge girders from falling, and to provide ductil for reinforced concrete piers.

Footings and Rafts

Some of the case histories provide information on the behav of footings and rafts founded on unusual soils or roc Butcher and Marsland describe the measured performance a bridge abutment on weathered chalk. The chalk was rub and partly weathered with bedding and jointing; the joi were 10 mm to 60 mm apart and some were open up 20 mm and filled with soft remolded chalk and ch fragments. The abutment was founded on a footing sized a maximum bearing stress of 400 kPa. The measu movements of the footing were small (3 mm to 4 mm) a are explained by a substantial increase in shear modulus the chalk with depth. Plate load tests conducted at or n footing level provided a good indication of modulus near surface, but were a poor indicator of overall performance the abutment because they did not reflect the increase modulus with depth. Lateral earth pressures due to placem of a well-graded granular backfill against the abutment wall were measured. The measured value of the earth pressure coefficient was about 0.2, although considerable scatter in the data was evident.

Felio and Bauer also describe the measured movements of a bridge abutment. The abutment footing was founded on a 2.5-meter-thick pad of compacted granular fill. This new fill was placed on top of an old dense fill (clayey silt) with an average N-value of 52 and a thickness of 6 m. The footing had a maximum bearing stress about 200 kPa and settled about 8 mm. Approximately 1 mm of settlement was associated with strains within the newly placed granular fill, the remaining settlement (7 mm) was associated with strains within the underlying soils. Stress measurements on the abutment wall indicated an earth pressure coefficient of about 0.35. However, the stress cells required temperature corrections and the reliability of the interpreted measurements is of concern to the Authors.

LaGatta and Keller present the results of load tests on two large footings (about 7 m x 3.5 m x 1.3 m) founded on a 6-m-thick layer of non-plastic silt. The maximum total settlement of these footings under applied loads of about 80 kPa was 9 mm. Approximately two-thirds of the total settlement occurred upon initial load application. Two settlement prediction methods for footings on sand were applied to this case history, both are based on N-values. Peck's method overpredicted displacements by 100 percent and Meyerhoff's method overpredicted displacements by 30 percent.

Lutenegger, Remmes, and Handfelt describe the settlement performance of a raft foundation on partially saturated loess. The measured settlement under a surface load of about 70 kPa was 12 mm on first loading and 33 mm six years after load application. The long-term settlements were found to be relatively consistent with predictions made based on the results of one-dimensional consolidation tests, using volumetric strains at the end of 24-hour load increments. The short-term settlement of 12 mm was consistent with settlement predictions based on stress path triaxial tests. The Authors did not compare the laboratory values of coefficient of secondary compression with field measurements of the rate of secondary compression. Such comparisons would have been useful. The Reporters noted that the imposed load approached the preconsolidation pressure of the loess. This makes predictions of settlement more difficult and also could have an important effect on the rate of secondary compression.

Bauer presents thirteen years of data on the measured settlement of five footings founded on overconsolidated clay. The settlements during the construction period varied from 6 mm to 11 mm and the total settlements after thirteen years ranged from 11 mm to 19 mm. The applied bearing stresses ranged from 125 kPa to 270 kPa. Measurements of settlement with depth suggest that ninety percent of the settlement takes place within a depth equal to twice the width of the footing, as compared to predictions from elastic theory that suggest that about seventy percent of the settlement should have occurred within this depth range.

Winter and Chung discuss the measured behavior of two mat foundations and compare measured settlements to those predicted from an analytical model that considers an elastic plate supported on an array of independent vertical springs that represent the soil. A major challenge in using this method is selecting the spring constants. The Authors estimate an average value of Young's modulus, E, for the formation and use a method proposed by Vesic to calculate the modulus of subgrade reaction. The analytical model is shown to produce about the right amount of total settlements and, in a general way, to follow the observed pattern of settlements. However, the Reporters are concerned that the agreement between measured and predicted movements may be fortuitous because the procedure used by the Authors for finding a weighted average value of modulus is open to question. The Reporters believe that the following approximate procedure could be used for computing an average modulus for a layered profile. Assume the layered profile is subjected to one-dimensional compression under a stress increment that is constant with depth. The layer settlement, S_i, can be computed from the layer thickness, H_i, the layer constrained-modulus, D_i, and the layer stress change, q, as follows: $S_i = \frac{q}{D_i} H_i$. The total settlement, S, can be determined by adding up the contributions of the various layers:

$$S = \sum S_i = \sum \frac{q}{D_i} H_i \qquad \text{Eq 1}$$

The total settlement, S, can also be expressed in function of the total thickness, H, and an equivalent modulus D:

$$S = \frac{q}{D} H \qquad \text{Eq 2}$$

where $H = \sum H_i$

Equating Eq 1 and Eq 2 suggests that:

$$D = \frac{H}{\sum \frac{H_i}{D_i}} \qquad \text{Eq 3}$$

The Authors have assumed that the equivalent modulus can be computed as

$$D = \frac{\sum H_i D_i}{H} \qquad \text{Eq 4}$$

Using Eq 4, the Authors report an average modulus of 350 tsf for their Case 1. The Reporters calculated an average modulus of 130 tsf using Eq 3. The difference between average moduli calculated by Eq 3 and Eq 4 is small when there is a small variation of modulus with depth, but can be large for significant variations in modulus with depth. Eq 4 always gives an average modulus that is too high because it understates the contribution to settlement of softer layers.

Abo-El Magd, Hosny, and Mashhour describe the structural distress of a small one- to two-story masonry villa founded on footings and underlain by medium dense to dense sands and stiff clays. The Authors do not report the magnitude of movements but suggest they were very small. It is suggested that changes in construction techniques and design details can reduce the problem. The Reporters were surprised that a structure that is commonly built in the region would be so settlement sensitive. The general performance of similar structures would be of interest in that regard.

Deep Foundations

Fifteen papers deal with the predicted, measured, and/or allowable performance of deep foundations. The special lecture by Hansbo and the paper by Olson, Dennis, and Winter are of particular interest and will be discussed first.

Hansbo discusses a pile-raft foundation system that has been successfully used in Sweden for buildings founded over deep deposits of soft clay. The essence of the concept is that the piles are loaded to failure (the creep failure load) and the raft exerts stresses less than the preconsolidation stress of the clay. Four case histories are used to illustate the peformance of structures founded on pile-raft foundation systems. One case history is particularly important because it compares the performance of two similar buildings; one supported on conventional friction piles with a Safety Factor of 3, the other supported on a pile-raft system. The measured data show that the settlements of the two buildings are about the same two years after construction. Future consolidation settlements are unknown and the pore pressure data presented in the paper are not detailed enough to draw any conclusions about future settlements.

Hansbo is to be congratulated for the implementation of this innovative foundation system. However, the Reporters would have benefited from a more explicit discussion of how the pile-raft system works. It appears to the Reporters that a potential drawback of the new system is that the settlement of the structure is sensitive to the capacity of the piles, whereas the settlement of a similar structure on conventional friction piles is insensitive to the capacity of the piles. This is illustrated in Figure 2. A simplified distribution of the

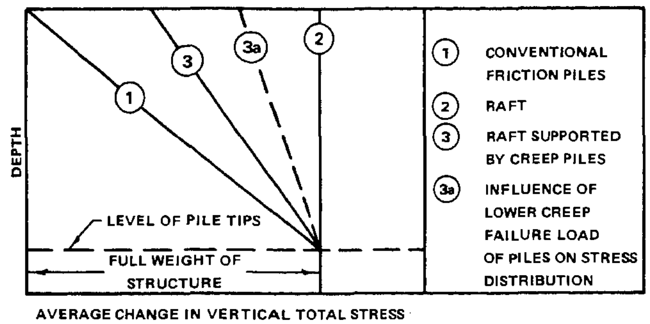

FIGURE 2 - COMPARISON OF STRESS CHANGES FOR CONVENTIONAL FRICTION PILES, RAFTS, AND PILE SUPPORTED RAFTS

change in vertical total stress with depth is shown for conventional friction piles (Curve 1), a raft (Curve 2), and a raft supported by creep piles (Curve 3). The reduction in settlement associated with the use of conventional pile foundations rather than a raft results mainly from the fact that the stress changes within the clay are significantly reduced. The same can be said for the raft supported by creep piles, but the stress reduction with creep piles is not as large as that with conventional friction piles. The potential problem with rafts supported by creep piles is that the stress changes on the clay are sensitive to the engineer's ability to predict the creep failure load. For example, if the creep failure load were only one-half of that estimated, the stress changes on the clay would change from that shown by Curve 3 to that shown by Curve 3a. On the other hand, if the pile failure load were one-half that estimated by the engineer, there would be virtually no change in stress distribution for the friction pile foundation designed for a Safety Factor of 3.

Olson, Dennis, and Winter present the results of an extensive comparison between measured pile capacities and pile capacities calculated using several available methods. Over 5000 load tests were examined and only about 1000 were included in the data base. The main reason for rejection was a lack of soils data or the application of loads much less than the plunging failure load. Calculations of pile capacities were made for piles in clay using five existing methods, and also a sixth method developed by the Authors. Three of the original five methods are based on empirical correlations with average undrained shear strength, the difference between them being the rules for determining the factor, alpha, that relates undrained shear strength to shear stress at the pile face at failure. Two of the methods are based on empirical correlations with average undrained shear strength vertical effective stress. The empirical coefficient in case is called lambda, and the difference in the methoc related to the rules for estimating lambda.

The data base used for testing the methods inclu sixty-seven cases where untapered full displacement p were installed in clay and loaded to failure in compress Comparisons of measured to calculated capacities sho that all five methods predicted the measured capacities wi ten percent (on average). The new method proposed by writers produced an average ratio of predicted to meas capacity of unity. Unfortunately, all six prediction met produced values of the capacity ratio (calcul capacity/measured capacity, Qc/Qm) that scatt considerably about the mean. The capacity ratio was fo to be log normally distributed and the minimum stan deviation of ln Qc/Qm was found to be 0.3. This imp that in about one case out of three, the value of Qc was less than 0.50 or greater than 1.35. Similarly, the imp value of Qc/Qm that would be exceeded one time out thousand would be 2.0. This corresponds to a factor of sa of 2, but does not account for the uncertainty in load. uncertainty in load (ie, the potential for overloading) w tend to increase the factor of safety required. Thus, not surprising that the Swedish Building code requires a fa of safety of 3 for friction piles when capacities are calcul from static formula and not verified by load test.

Olson, Dennis, and Winter also consider piles in sand, look at the effects of pile length, taper, and type of loac This comprehensive study of load test data shows quite cle that the widely used empirical methods for prediction of capacity are not very reliable, and allows the uncertaint be quantified.

Xu, Liu, Shi, and Xin describe the measured performanc single piles in sand when subjected to vibratory loads. T found that the creep rate during sustained vibrations stable (decreasing with time) or unstable (increasing time) depending on the magnitude of the static load dynamic load. A method for using the special load test design is discussed.

Negative skin friction was of concern in at least four pap Lee and Sithichaikasem discuss pile design for twenty-two-story hotel in Bangkok. Significant gro movement above the pile tips was expected because regional subsidence and negative skin friction loads w result from this movement. The designers decided to us bituminous coating to reduce negative skin friction to va of 0.1 t/m^2 or less. A pile load testing program undertaken to verify that the piles would have a Safety Fa of at least 1.5 considering building loads and negative friction loads, and considering pile capacity derived from s below the depth where the friction changes from negativ positive. An additional guideline was that the allowable should not be greater than approximately one-third of ultimate capacity measured in short-term load tests. Th guidelines were achieved using the bitumen coated piles.

Clayton, Milititsky, and Carvalho discuss the foundation de for a large mill complex. The pile foundation for this com had to overcome a myriad of problems: a settlement sensi heavily-loaded structure was to be founded on piles dr through 9 m of clay fill to a 3-m-thick layer of limest underlain by a thick layer of stiff clay. The technical prob of most concern was the potential for piles punching thro the limestone. The tip stress beneath the piles would about 5 MN/m^2 at nominal working loads, a very large st when compared to an unconfined compressive strength to 5 MN/m^2 in the upper portion of the rock layer.

Eight piles were load tested to 165 t, 1.5 times the nominal design load. The load that reached the pile tip during these load tests was estimated to be about 100 t because of load carried by the overlying clay fill. If settlements of the clay fill occur and produce negative skin friction, the Authors estimate that the negative skin friction load for a pile within a large pile group would be about 35 t. Thus, the tip load of 100 t applied during the load tests is considerably less than the expected tip load of 145 t with negative skin friction. Three additional load tests were conducted with maximum applied loads of 250 t. The load applied to the pile tip for these tests is estimated to be about 185 t, or 40 t greater than the expected tip load of 145 t. Thus, the applied tip load during these tests was about thirty percent higher than the expected maximum tip load.

This structure has performed very well, but the Reporters were stunned by the number of ways that unsatisfactory performance of this foundation system could have occurred. A discussion of the reasons for choosing this foundation system over other alternatives would have been a valuable addition to the paper.

Bhandari, Soneja, and Sharma discuss a field experiment to estimate negative skin friction for a bored pile in clay. A 28.5 m long pile, 60 cm in diameter, penetrated about 18 m of soft clay, 9 m of very stiff clay, and was socketed into weathered rock for about 0.5 m. Ground settlements were induced by building a fill around the pile. The rate of ground settlement was accelerated by installing 30 cm diameter sand drains on 2 m centers. Pore pressure measurements with depth indicated zero pore pressure response below a depth of 8 m under an applied surface load of 3.2 t/m^2, but significant pore pressure response at these depths occurred when the surface load was increased to 4.8 t/m^2. The Authors suggest that this pore pressure response can be used to estimate the depth of soil generating negative drag, and speculate that the upper 3 m of lightly overconsolidated clay fill may be acting as a natural raft, thereby preventing stress transfer below 8 m depth under a surface load of 3.2 t/m^2. This raft action was apparently broken when the load was increased to 4.8 t/m^2. The Reporters believe that this unusual behavior may be due to the soil reinforcement provided by the sand drains that are 30 cm diameter and 2 m, or less, apart. These sand columns may have been able to carry the applied load of 3.2 t/m^2, but failed under a load of 4.8 t/m^2.

DeBeer, Wallays,and Goelen discuss the implications of combined lateral loading and negative skin friction on piles. This is a common load situation for bridge abutments and for storage yards with crane rails. The Authors point out that both lateral loads and negative skin friction loads should be accounted for in design, and that the lateral loading often controls the design.

Barton, Parry, and Finn investigated the performance of piles in sand under cyclic lateral loading using the large centrifuge at Cambridge University. Extraordinary agreement was found between behavior measured in the field at Mustang Island and behavior measured in the centrifuge model. The prototype and model performances were compared after ten cycles of lateral loading because experience has shown that the behavior during repeated lateral loading stabilizes after five to ten cycles of loading. The Reporters support the Authors' conclusion that centrifuge testing has a major role to play in providing case history data under carefully controlled, fully monitored conditions. However, the extraordinary agreement between model and prototype behavior demonstrated for lateral loading of piles in sand cannot be expected in general. The state change of the sand around the pile caused by cyclic loading is nicely modeled in the centrifuge and probably has an important effect on pile response. On the other hand, the state changes in sand during pile installation is a dominant factor for the response of piles during axial loading, and this state change may be very difficult to model in the centrifuge.

Briaud, Pacal, and Shively compare measured to predicted load-deflection behavior for drilled shafts installed to support transmission towers. The shafts were 10 ft to 15 ft deep and ranged from 25 inches to 36 inches in diameter, and were load tested in tension and under lateral loads. The investigation at the three sites included pressuremeter tests and cone penetrometer tests. The Authors report that predictions of behavior were made before the load tests were run. The comparison of measured to predicted load-deflection behavior under lateral loads is very good. Similar comparisons for tension loading show considerable discrepancies.

Blight describes the potential problems associated with swelling of desiccated deposits of alluvium and siltstone in the presence of water. The influence of ground swelling on deep foundations was a particular concern. Predictions of ground swelling were made based on estimates of the change in effective stress. The uplift load that could be transferred to piles was also predicted and a pile design that uses a layer of vermiculite to isolate the pile and reduce the uplift force was developed and implemented. Voids were left underneath floor slabs (by means of collapsible cardboard forms or undermining) to prevent uplift forces on the floor slabs. The Reporters consider this foundation design to be prudent because satisfactory performance of the piles is assured, virtually independent of the actual amount of heave that occurs. Evaluation of the methods for prediction of the magnitude and rate of heave must await the measured field behavior.

Kaderabek, Barreiro, and Call describe a thirteen-story building in Florida that was founded on short piles, but underlain by two layers of very loose silty sand, one about 10 ft thick, the other about 6 ft thick. Five borings were made at the site and these limited data suggested N-values of 2 to 3 for these sand layers. Estimates of settlement made after construction using several published methods for estimating modulus from N-values, and several methods for predicting settlement from modulus, varied from 2 inches to 8 inches. The measured settlement was about 1.8 inches and some cracking of shear walls occurred. This case history would be more valuable if additional data were available on the problem layers. The Reporters have found that, for apparently the same soil conditions, a loose sand layer can settle 3/4 inch in one instance and 3-1/2 inches in another under the same surface load. Consequently, preloading of sites in Florida under a load greater than that of the building is an economical way to improve deep layers of sand and prevent large settlements under the weight of the building.

Engeling, Hayden, and Hawkins describe the installation and load testing of Raymond concrete cylinder piles in the Arabian Gulf. The project was plagued by difficulties with pile installation associated with unexpected subsurface conditions, problems with predrilling, and piles reaching refusal at shallow penetrations. The predrilling problems were solved by direct circulation drilling using heavy drilling mud. It was so effective, in fact, that concerns developed regarding lateral load behavior and tension capacity of the piles. There was also concern about whether high driving resistances at low penetrations could be relied on in carbonate materials. An extensive load testing program demonstrated that wave equation analysis provided reasonable estimates of pile capacity and that installation procedures could be developed to achieve the required tension capacity and lateral load capacity.

Arcones and Soriano report the results of driving piles into a stratum of sand that had been improved by vibrocompaction before pile driving. The vibrocompaction was required to avoid the potential for liquefaction. The piles were provided to control settlements of a heavy chimney and were driven to a depth of about 14 m below the original ground surface. Settlement predictions assumed the structural load was applied on a raft at the base of the piles and a settlement reduction factor of 2 was used to account for embedment. Predictions of settlement made before construction overestimated settlements by twenty percent. Unfortunately, the method used to estimate the modulus for the sand is not reported. The pile driving to a predetermined depth was difficult because of the variability of the soil improved by vibrocompaction.

Lane presents a case history illustrating how instrumented caisson load tests can be used for design. The caissons were load tested to 2.5 times design load and it was verified that, at the design load, virtually all the load was carried in skin friction. The settlements under working loads are small and the ultimate capacity is very large because base resistance keeps increasing with displacement. This comprehensive paper deserves study by everyone involved in caisson design.

Chung and Cundy discuss the caisson design for a 305-m-high chimney. The ground conditions included glacial outwash over sandstone. The core recoveries within the sandstone were always 100 percent and, once a thin weathered zone was penetrated, RQD values generally ranged from 50 percent to 90 percent. A typical unconfined compressive strength of the rock was 70 MPa. The Authors describe how design values for allowable side shear and end bearing were estimated from a review of published load test data. They also describe the field installation techniques, which included inspection of every rock socket after drilling and dewatering. The Reporters believe this paper is an excellent example of careful and prudent design and attention to construction details that are essential for the successful use of high-capacity caissons.

Tanks and Silos

Six papers consider the performance of tanks and silos. Zhang has contributed a paper with an abundance of data on measured performance of tanks at a soft clay site in China. Twenty-two of the tanks were 12 m in diameter, 9 m high, and had a cone roof. The other eleven tanks had dome roofs, were 12 m to 30 m in diameter, and were 9 m to 13 m in height. The load of the oil tanks was 11 t/m^2 to 15 t/m^2. The undrained shear strength of the clay is reported to be about 1.5 t/m^2. Thus, the tank loading ranged from about seven to ten times the undrained shear strength of the clay. The tanks were preloaded with water prior to being put in service and flexible connectors were used during preloading. Water preloading was accomplished in three stages. The duration of preloading was consistently fifteen days for the 13 m high tanks and the average long-term tank settlement was 90 cm. The duration of preloading for the 9 m high tanks varied considerably, but a duration of 45 days is a representative value. The average long-term settlement of the 9 m high tanks was 75 cm. All tanks performed satisfactorily and relative tilt and differential settlement are presented for all tanks. Data are available on the differential settlements before and after the tank farm experienced an earthquake that had a magnitude of 7.8 on the Richter scale. In general, the earthquake had little or no effect on the tanks.

Gazioglu and Withiam describe the performance of a floating roof 300-ft-diameter steel tank, 33 ft high. Six tanks were constructed next to the Mississippi River in St James, Louisiana and one of the six tanks experienced excessive differential settlements under one portion of the ringwall during water preloading. The preloading was stopped and detailed investigation revealed that there was a deposit more compressible soil beneath the area of the tank t settled excessively. Settlement estimates showed expected total settlement of the tank would lead unacceptably large differential settlements. Consequen the tank was placed in limited service and only used temporary storage of product.

Wei's paper presents the results of pore press measurements beneath a large oil tank founded on a th deposit of soft clay. His interpretation of the data consistent with those of earlier researchers; ie, the exc pore pressure at a point beneath a loaded area increa linearly with the increasing surface load during undrai loading until a certain critical load is reached. Then, induced shear stress at that point reaches the shear strength of the soil and local shear failure occurs. Thereaft there is a pronounced increase in the rate of pore press buildup with applied load. In the Reporters' opinion, We major contribution is the analysis of pore pressure data dur second loading, and then after many cycles of loading. T change in pore pressure response during undrained loadi more precisely the change in critical load for ea piezometer, can be used to track the effectiveness preloading for increasing soil strength.

Ahmed describes the soil improvement methods used construct four 270-ft-diameter floating roof tanks at a s soil site adjacent to the Mississippi River. The tanks we 32 ft high and applied a surface load of 2000 lb/ft^2. Alidra were installed on 8-ft centers in a peripheral band extend 24 ft inside and 16 ft outside the edge of the tank. Thr of the four tank sites were preloaded with an earthfill pr to tank construction and water loading. The fi measurement data contained in the paper do not provide clear picture of ground performance and tank performar through the various phases of site improvement and ta construction. It is suggested that pore pressures equal the applied surface loading are tolerable near the center the tank, and that limiting the pore pressures to 75 perce of the applied surface loading near the edges of the ta would preclude bearing capacity failure. However, t rationale for those statements is not presented.

Ozaydin and Inan discuss the measured performance of fi tanks ranging in diameter from 75 m to 100 m with a heig of 15 m to 17 m. The measured settlements during wa load tests were 2 cm to 7 cm, as compared to predictio of edge settlements of 25 cm to 45 cm that were made others before the test. Moreoever, the rate of settleme was much faster than predicted. The Authors reanalyzed t data, using soil parameters that would match the measur behavior and concluded that the original estimates of s properties were not correct. However, information on s properties is not provided and no explanation of why t original estimates of soil properties were so different fro the actual values is offered.

Saye describes the settlement behavior of grain tanks alluvial soils at two sites in Iowa. Accurate predictions the settlement of the three tanks requires defining the stre history of an alluvial soil deposit with a desiccated cru The importance of using a simple geologic model to evalua the implications of desiccation on stress history is explaine and a number of methods for determining the lowest wat level in the past (the relicit water level) are discussed. T Reporters noted that Saye's concept of uniform r overconsolidation below the relicit water level is valid f hydrostatic groundwater conditions, but would have to modified somewhat for the case of upward steady sta seepage from a deeper more pervious zone. It should al e noted that a break in the load settlement behavior during ndrained loading is generally considered due to local yielding nd contained plastic flow, rather than due to load exceeding he preconsolidation pressure.

DJACENT STRUCTURES

xperience has shown that it is very important for the oundation engineer to consider the possible interaction etween the new building and adjacent structures. Ignoring uch interaction can lead to serious damage to the existing tructures and even to problems with the new construction. s shown in Figure 1, five papers are included in Theme One hat deal with this problem. Four of these papers discuss he possible adverse effects of new construction on existing tructures and will be addressed first. The last paper escribes a case history where the existence of a nearby tructure had detrimental effects on a new building.

uong describes how construction of a new sewer had etrimental effects on a nearby twenty-year-old building. he building was founded on fill over alluvial deposits verlying marl and a layer of residual gypsum. The deep ayers included limestone and sands. The construction of the earby sewer required the installation of a large well within 0 m of the building to dewater the construction site. Several racks appeared in the building as a direct result of the ewatering operation; cracking stopped as soon as the ewatering was suspended. The intense pumping for the onstruction of the sewer is believed to have generated roundwater movement in the gypsum rich soils underlying he building, dissolving the gypsum and generating settlements f the building. The Author concludes that it was unwise to roceed with the dewatering program in an urban area, ithout special provisions to protect the existing building hen it is well known that gypsum is soluable in water and hat cavities in gypsum rich soils can progress rapidly with roundwater movement and can lead to disastrous results.

hummar discusses the construction of a five-story building n a raft foundation within 1.5 m of an existing three-story uilding on strip foundations. The subsurface investigation evealed 2.5 m of clayey sand overlying 11.5 m of soft silty lay. One year after construction of the new buildings, the xisting building was observed to have settled about 100 mm n the side of the 5-story building and to have tilted onsiderably. The Author concludes, after settlement and earing capacity analyses, that construction of the new uilding induced additional consolidation settlement of the lay and also generated creep of this material in a zone here the loads from the two buildings are superimposed and xceed fifty percent of the calculated ultimate strength of he clay.

iancia and Horn present a case history where extensive nvestigation, careful design, instrumentation, and onstruction supervision led to the successful completion of large excavation in a crowded city environment without etrimental effects on neighboring structures. Construction f a twenty-six-story steel-framed structure in New York ity required the demolition of a low-rise building with a 0-ft deep basement that occupied the site. Temporary upport was required during demolition and until the new oundation system was constructed, to protect three adjacent ity streets, a fifty-three-story office tower, and two nearby ctive subway tunnels. The subsurface conditions consisted f a surficial layer of fill overlying mica schist rock. Three f the four foundation walls of the existing building were to emain as part of the new building. Probe holes were drilled hrough these walls to evaluate the subsurface conditions and evelop earth pressure envelopes for the design of a temporary aker system. An optical monitoring system and borehole extensometers were set up to monitor wall movements throughout the demolition operations. Measured deflections of the walls varied from 0.001 inch to 0.013 inch. The existing foundation wall near the fifty-three-story building had to be removed. Probe holes were drilled to determine whether the rock was in contact with the wall, rock anchors were installed, weepholes were drilled, borehole extensometers were installed to monitor movements, "windows" in the wall were removed to allow for detailed geologic mapping of the rock and provide the basis for stability analyses of the rock face. Thereafter, the wall was demolished in stages with continuous geologic mapping and monitoring of the extensometers. The wall was successfully demolished and the movement recorded varied from 0.001 inch to 0.004 inch resulting in no effect on the fifty-three-story building. The Authors also describe the results of a comprehensive geological investigation which showed that the foliation and other discontinuities of the rock were favorably oriented to permit the construction of heavily loaded footings 40 ft above the crown of one of the active subway tunnels.

Dugan and Freed address the problem of ground heave outside the construction site due to pile driving. The Authors draw general conclusions regarding the factors influencing ground heave during pile driving based on data from nine case histories in the Boston area where end-bearing piles were driven through a thick deposit of insensitive soft to stiff clay to a layer of hardpan or to bedrock. Some of the major conclusions are as follows:

- The magnitude of heave is directly proportional to the volume of clay displaced by the driving operations.
- The lateral extent of ground heave is about equal to the depth to the bottom of the clay.
- The amount of ground heave is inversely proportional to the existing vertical stress. Buildings and other above ground structures experience less heave than the ground surface.
- The ground heave increases in the direction toward which the piles are sequentially driven.

Other factors that can influence the ground heave include pile installation procedures, clay sensitivity, excavation depth, and the presence of granular layers. The Authors conclude that ground heave due to pile driving is a temporary condition; the displacements and excess pore pressures generated by the driving operations will eventually cause consolidation of the clay and settlement. The Authors tentatively suggest, based on the data collected, that the net settlement resulting from pile driving is about equal to the expected ground heave.

The last paper presented in this category deals with the construction of a one-story annex to a three-story building. The subsurface conditions, as described by El-Sohby and Mazen, consist of 1 m of fill, overlying 5.6 m of very soft clay, over 2.6 m of soft clay, underlain by sand that becomes coarser and agglomerated with depth. The three-story building was founded on piles; the annex was constructed on inverted T-strip footings with foundation pressures varying from 9 to 21 kN/m^2. Settlement of the annex building was monitored for five years and revealed that the building settled from 60 mm to 170 mm along the connection with the three-story building and from 100 mm to 250 mm along its free edge, away from the three-story building. After analyses, the Authors conclude that the settlements are the result of the consolidation of the clay under load. The differential settlements are due to the combination of varying foundation loads and lateral confinement provided by the piles of the three-story building that restrict the lateral displacement of the clay, thereby restricting consolidation and settlement along the connection between the two buildings.

INNOVATIVE FOUNDATION SYSTEMS

The development of new or improved foundation elements or systems is a challenging and creative part of foundation engineering. Innovation is usually prompted by a need to solve an unusual problem. In general, little experience is available for an innovative foundation system and, as a result, the risk of unsatisfactory performance is relatively high. The use of innovative foundation systems requires a willingness on the part of the Owner to support the additional testing required to investigate the suitability of the new foundation system, and acceptance by the Owner of a greater than normal risk of unsatisfactory performance. The increased risk of poor performance for innovative foundation systems should be borne by the Owner because he receives the expected benefit from the innovation.

Eight papers identified in Figure 1 consider innovative foundation systems.

Handa describes the foundation systems provided for the Taj Mahal and the Qutb Minar in India. These structures were built in about 1650 and 1200, respectively, and are interesting examples of innovative foundation engineering for very old monumental structures.

Hansbo describes the use of a pile-raft system in which friction piles in clay are loaded to full capacity. This innovative foundation system achieved substantial reductions in the settlement of the building as compared to a raft foundation alone. In fact, the total settlements appear to be similar to those expected for conventional friction pile foundations and the differential settlements may be less than conventional friction piles. This innovative foundation system offers the hope of better performance and significant cost savings. It has been applied successfully for three buildings in Sweden and the risk of unsatisfactory performance appears to be relatively small when weighed against the substantial cost savings in foundations.

Blight describes field testing of methods for reducing uplift on drilled shafts penetrating desiccated soils that are expected to swell as they become saturated. It was verified by field testing that filling the annular space between the shaft of the caisson and an outer casing with vermiculite was an effective bond breaker. The drilling of stress relief holes around the caisson was attempted and found to be ineffective in reducing uplift forces. This is a good case history showing how careful testing can lead to development of an innovative foundation concept for a specific project.

Felio and Bauer describe the performance of a bridge abutment founded on compacted fill. This was a demonstration project to show that abutments founded on fill can perform satisfactorily. Although not new in many foundation applications, the concept of supporting footings and walls on compacted fill, rather than piles, is new to bridge designers and they are extremely cautious about its application.

Wilson, Stomer, and Girault describe foundation engineering for a hospital facility in Mexico City. The foundation system finally adopted included footings bearing on basaltic lava and designed for an allowable bearing pressure of 200 t/m^2, and footings bearing on coarse sand fill and lava fragments, densified by dynamic compaction, and designed for an allowable stress of 30 t/m^2. Both foundation systems are unusual and the combination of the two at a single building is even more so. However, the Authors describe a very cautious and comprehensive program of field investigat and construction inspection and demonstrate c convincingly that the risk of unsatisfactory performance very small.

Bauer describes a building supported on shallow foundat bearing on overconsolidated clay. Several special proced were used in order to produce a foundation design differential settlements within tolerable limits. The spe procedures included:

- o Decreasing the effective bearing area of lightly lo exterior walls to increase settlement of walls minimize differential settlement.
- o Placing a 5 inch thick layer of lightly compacted fill beneath some intermediate width strip footing produce 0.8 cm to 1.3 cm of movement on first loa of the footings.
- o Delaying construction of a section of the lowest f system so that lightly loaded elements could constructed last.
- o Regulating construction procedures so that shoring l were applied directly to footings.
- o Founding footings at different depths to ach compatibility of settlements.

Bauer's procedures may have been effective but Reporters' impression is that procedures such as these in an ability to predict and control the settlement of indivi foundation elements that is not achievable in practice.

Newman and DiGioia used adjustable columns to accommoc differential settlement. Their building was on stilts so the base of columns were readily accessible and adjustm could be made without interference with building operati Once the Authors decided to incorporate column adjustme they also decided to increase substantially the potential large column settlements by increasing the allowable bea stress for footings from 2 $kips/ft^2$ to 8 $kips/ft^2$. Exces movements did occur at several columns and it was fo that it was not possible to jack the columns all the way b to their original position without overloading them beca of the framing action of the building. However, the buil performed satisfactorily and the adjustable columns allo considerable cost savings.

Wang and Yuan describe a method for correcting the til of columns supporting crane rails, roofs, etc, in indust buildings. The procedure involves providing a correct moment to the footing by installing a collar and moment on the footing, and applying load to the moment arm jacking against a reaction pile. The system has reporte been used successfully on a number of projects in China

OTHER ISSUES

Ranjan, Prakash, Saran and Singh discuss a case in wh review of an existing design revealed that Safety Fact were not adequate. The redesign was completed and reme action implemented before problems developed. Reporters believe that this case presents a classic exam of the value of Peer Review as a quality control meas One of the best ways to consistently provide sound foundat engineering recommendations and decisions is to have recommendations and decisions reviewed by a qualif professional who has not been actively involved in the proje Many consulting engineering firms have adopted Peer Revi as the heart of their quality assurance programs.

Five of the papers included under Theme One are probably more relevant to other themes at this International Conference. The guest lecture by Iwasaki contains a wealth of experience on the behavior of bridges during earthquakes and should be considered carefully by those interested in Theme Five: Earthquake Engineering. Taylor and Joseph's paper considers the influence of slope instability on a power plant and was considered herein under the topic unexpected subsurface conditions. This paper is also relevant to Theme Three: Dams, Embankments and Slopes. Bhargava, Nath, Kapoor and Singh's paper considers the influence of slope stability on power plant construction and was also considered under the topic of unexpected subsurface conditions. This paper is also relevant to Theme Three. Two other papers in Theme One are relevant to Theme Three and have not been discussed as yet in this General Report. A brief discussion of these papers follows.

Sheng describes the measured movements of two excavations in clay for dry dock construction in China. The construction sequence for the first excavation was excavation under water using a dredge, construction of a cofferdam, dewatering of the slopes with well points and then pumping out the excavated area. The second excavation was accomplished by dewatering using well points, followed by excavation in the dry.

The second excavation procedure, while unconventional, allowed the use of steeper slopes and the slopes experienced smaller movements. Finite element analyses were made for the second excavation case. The analyses gave a pattern of deformations consistent with those measured but the predicted movements were smaller than those measured.

Jamiolkowski and Lancellotta compare the measured pore pressure beneath an embankment on clay to the pore pressure predicted by several analytical methods. The pore pressures measured during construction were 82 to 92 percent of the estimated change in vertical total stress during construction. The Authors conclude that predictions using the modified Cam-Clay soil behavior model, which suggests the undrained pore pressures are 117 percent of the applied vertical stress, are probably most correct and proceed to estimate values of coefficient of consolidation. These calculations were made despite the fact that the piezometers did not show the pore pressure dissipation after construction to be as expected. The Reporters believe that the measured pore pressures may not provide a reliable basis for evaluating prediction methods for undrained excess pore pressures, or for predicting rates of consolidation.

LIMITATIONS OF THIS REPORT

This General Report attempts to organize the fifty-four papers of Theme One in a useful way and to present a brief discussion of each paper. Both Reporters read each paper and reviewed the entire report. Nevertheless, it is possible that some of the papers have been misunderstood, or that errors were made in transposing information from the Papers to this General Report; particularly considering the limited time available for report preparation.